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# TRANSACTIONS.

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No. 810.

## RECENT TESTS OF BRIDGE MEMBERS.

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PRESENTED AT THE ANNUAL CONVENTION, 1897.

WITH DISCUSSION.

During the past two years the author has made a number of tension tests on bridge members other than eye-bars. These tests, while not exhaustive, are in a channel outside of the usual run, and it is believed that the results obtained furnish information of a kind likely to be of general interest and value. The investigations are divided into six series, *A*, *B*, *C*, *D*, *E* and *F*, of which *A*, *B* and *C* pertain to the strength and value of built-up tension members, *D* to the net area required back of pin holes in plates having sheared and planed ends, *E* to the tensile strength of single angles having ends riveted to connection plates, *F* to the strength of steel which has been worked partly hot and partly cold. These different series will be discussed in regular order.

*Series A, Tests 1 to 10.*—These tests were made with the view of ascertaining to what extent the individual pieces of built-up members

will act together while under a tensile strain, the efficiency of different styles of webs or lacing, and whether the results, as compared with specimens, make a better or worse showing than those obtained from eye-bars.

There are five designs and two tests of each design, the detail drawings of which are given in Fig. 1. End connections were designedly made strong enough to insure rupture in the body of member, but attention is called to the eccentric attachment of pin plates to pieces 1 and 2, and to the more direct attachment to the others. In members 1, 2, 5 and 6 the rivets are so arranged that but one hole in each angle need be deducted in order to arrive at the net section, but in pieces 3, 4, 7, 8, 9 and 10 the rivets stagger by less that $\frac{3}{4}$ in., thereby reducing the gross section of each angle by the area of two holes, or else requiring this area to be determined by a zigzag line passing through the nearest holes. The rivets used were $\frac{1}{2}$ in. in diameter, holes were punched $\frac{9}{16}$ in. in diameter and assumed as $\frac{5}{8}$ in. in diameter in arriving at the net section. The ends of plates were simply sheared off.

The material was soft basic open-hearth steel rolled at Pencoyd for Tests 1 and 2, and by the Carnegie Steel Company for Tests 3 to 10. Pieces 1 and 2 were made and tested at Pencoyd, the others at Edge Moor.

Table No. 1 gives the results of specimen tests cut from the material used in the built-up members, the elastic limit having been determined by the drop of the beam and the elongation taken in a length of 8 ins.

TABLE No. 1.—Specimen Tests, Series A.

| | Tests 1 and 2. | Tests 3 to 10. | | |
|---|---|---|---|---|
| | Angles. | Angles. | Plates. | Average. |
| Elastic limit | 40 700 lbs. | 39 900 lbs. | 36 600 lbs. | 38 250 lbs. |
| Ultimate strength | 58 200 lbs. | 56 500 lbs. | 59 300 lbs. | 57 800 lbs. |
| Elongation | 25.25% | 26.20% | 25.60% | 25.90% |
| Reduction | 62.50% | 57.00% | 57.00% | 57.00% |
| Fracture | Silky. | Silky. | Silky. | Silky. |

Table No. 2 contains all data in connection with the design of members in this series, the percentages being based on areas obtained from assumed stresses as follows: Tension, 15 000 lbs.; bearing on pins and rivets, 20 000 lbs.; shearing on rivets, 10 000 lbs.

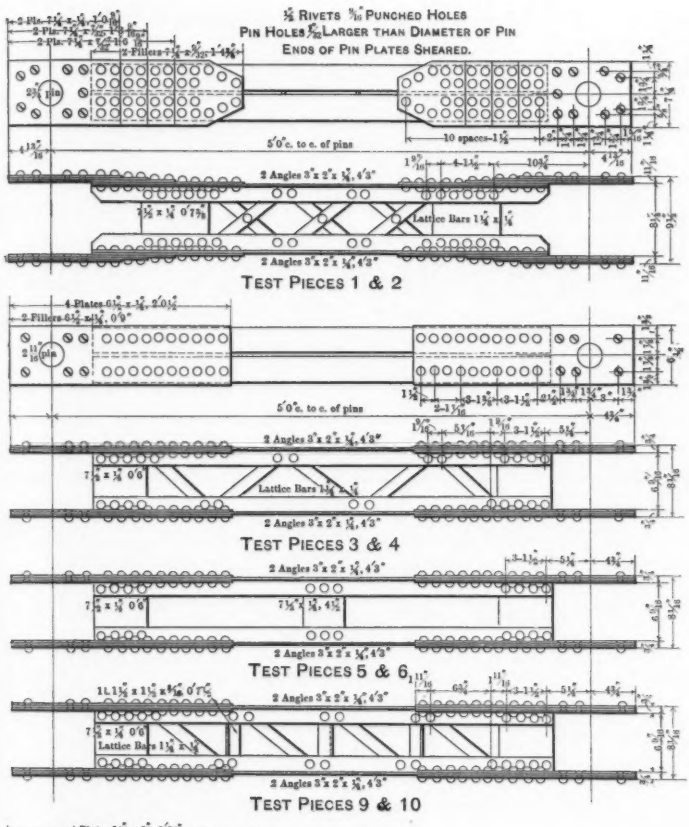
⅞ Rivets 15/16" Punched Holes
Pin Holes 1/32" Larger than Diameter of Pin
Ends of Pin Plates Sheared.
5'0"c. to c. of pins
10 spaces-1½"
2 Angles 3"x 2"x ¼", 4'3"
Lattice Bars 1¼" x ¼"
Test Pieces 1 & 2
5'0"c. to c. of pins
Lattice Bars 1¼" x ¼"
Test Pieces 3 & 4
Test Pieces 5 & 6
Test Pieces 9 & 10
Web Plate 7½" x ¼", 4'3"
Test Pieces 7 & 8

Fig. 1.

TABLE No. 2.—DATA RELATING TO MEMBERS OF SERIES A.

| Mark. | Area. | | Holes Deducted. | Percentage Excess. | | | |
|---|---|---|---|---|---|---|---|
| | Gross. | Net. | | Bearing on pins. | Net Area. | | Rivets. |
| | | | | | *Through pin holes. | *Back of pin holes. | |
| 1 and 2..... | 4.80 | 4.18 | 4 | 21.1 | 47.6 | 12.9 | 50.5 |
| 3 and 4..... | 4.80 | 3.78 | 8 | 41.5 | 50.0 | 34.6 | 76.2 |
| 5 and 6..... | 4.80 | 4.18 | 4 | 28.2 | 35.6 | 21.8 | 59.4 |
| 7 and 8..... | 6.68 | 5.34 | 10 | 34.2 | 41.6 | 36.3 | 37.5 |
| 9 and 1..... | 4.80 | 3.78 | 8 | 41.5 | 50.0 | 34.6 | 76.2 |

* The figures in these two columns represent the percentage of increased area over actual net area through body of member.

The elastic limit of the built pieces was determined by the action of the gauge and by observing the scaling of the members. The two observations agreed closely in all cases excepting Nos. 3 and 4 where the gauge determination was excessive by 9.1% for the former and 4.6% for the latter. The gauge determination is given in Table No. 3. Elongation is measured from back to back of pin holes.

TABLE No. 3.—RESULTS IN DETAIL OF TESTS IN SERIES A.

| Mark. | Elastic Limit. | | Ultimate Strength. | | Elongation. |
|---|---|---|---|---|---|
| | Total. | Per square inch. | Total. | Per square inch. | |
| | Pounds. | Pounds. | Pounds. | Pounds. | Inches. |
| 1..... | 136 000 | 32 536 | 199 000 | 47 607 | 1.31 |
| 2..... | 150 000 | 35 885 | 201 000 | 48 086 | 1.31 |
| 3..... | 182 000 | 48 148 | 200 900 | 53 148 | 1.53 |
| 4..... | 174 400 | 46 138 | 193 400 | 51 164 | 2.28 |
| 5..... | 159 200 | 38 086 | 200 900 | 48 062 | 2.41 |
| 6..... | 174 400 | 41 722 | 200 900 | 48 062 | 1.03 |
| 7..... | 231 300 | 43 315 | 269 200 | 50 412 | 1.53 |
| 8..... | 238 800 | 44 719 | 282 100 | 52 828 | 1.53 |
| 9..... | 185 800 | 49 153 | 205 000 | 54 233 | 1.41 |
| 10..... | 174 400 | 46 138 | 205 000 | 54 233 | 1.56 |

The average results, together with the percentage ratio of the developed strength to the strength of specimens, is given in Table No. 4.

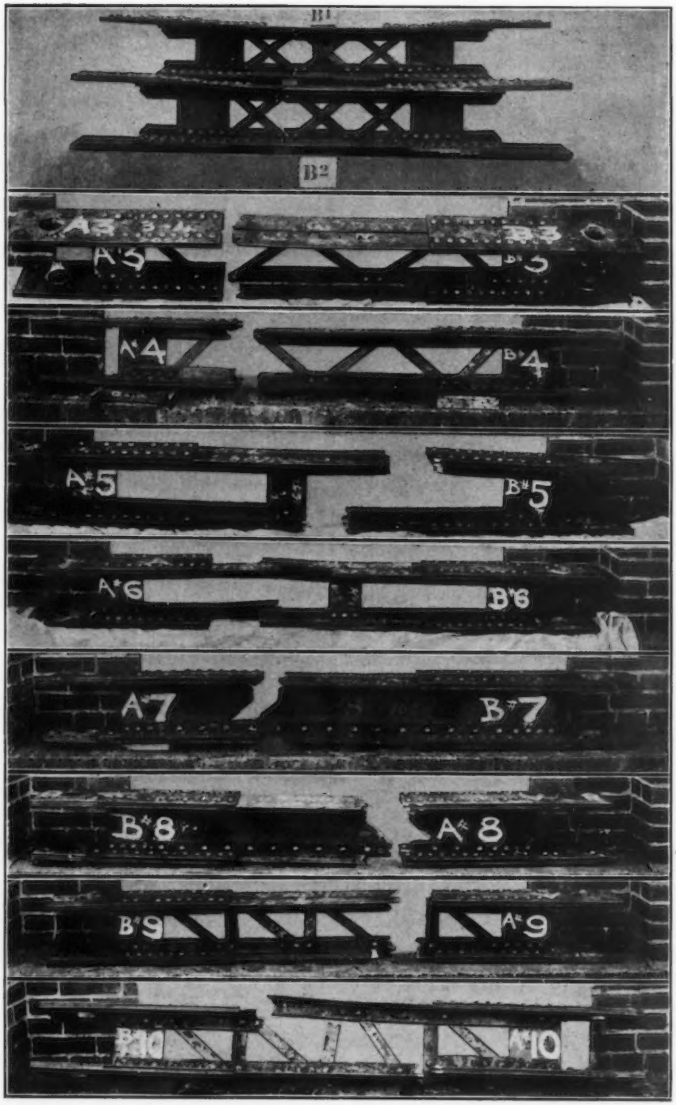
B1
B2
A3
B3
A5
B5
A4
B4
A5
B5
A6
B6
A7
B7
B8
A8
B9
A9
B10
A10

TABLE No. 4.—AVERAGE RESULTS AND COMPARISON WITH SPECIMENS.

| Mark. | Pounds Per Square Inch. | | Comparison with Specimens. | | Remarks. |
|---|---|---|---|---|---|
| | Elastic limit. | Ultimate strength. | Elastic limit. | Ultimate strength. | |
| 1 and 2..... | 34 211 | 47 847 | 84.0% | 82 2% | Double lattice. |
| 3 and 4..... | 47 143 | 52 156 | 118.2% | 92.3% | Single lattice. |
| 5 and 6..... | 39 904 | 48 062 | 100.0% | 85.06% | Batten plates. |
| 7 and 8..... | 44 017 | 51 620 | 115.1% | 89.3% | Solid web. |
| 9 and 10..... | 47 646 | 54 233 | 119.4% | 96.0% | Zigzag lattice. |

See Plate I for the appearance of the members after fracture.

The pins used in the first two tests were slightly bent, and the pin holes elongated $\frac{1}{16}$ in. In Tests 3 to 7 the pins were of soft steel, and were bent and sheared considerably, the bending amounting to as much as $\frac{5}{8}$ in. in a length of 15 ins. The bent pins distorted the holes, and as the pull was transmitted through links taking hold of the pins on the outside of the plates, the bending had a tendency to force these pin plates together. In Tests 8 and 10 the pins were case-hardened and bent but $\frac{3}{16}$ in., with no perceptible shearing and no distortion of the pin holes. Fracture was generally clean and silky, with an occasional granular spot about the rivet holes. The rivets remained tight, with no indication of shearing except where noted.

Observations taken during tests are as follows:

No. 1. The lattice bars drew the flange angles in to a noticeable extent, while the jaws bent inward at the ends. Fracture began at the edge of the inner leg on a line with the rivets in the first pair of lattice bars, the cross-section of the angles contracting noticeably at the critical line before pulling apart.

No. 2. This was a duplicate of No. 1; it broke in a similar manner and had similar accompanying characteristics.

No. 3. The lattice bars pulled the flange angles together and the jaws contracted, but all distortion was of a much less extent than that observed in Tests 1 and 2. Fracture began at the edge of the outer leg on a line with the rivet in the lattice bar near the tie plate. Rupture of the other angles followed an instant later, but they were bent outward, due to the uneven pull after the first fracture.

No. 4. This was a duplicate of No. 3, broke in similar manner and had similar accompanying characteristics.

No. 5. The flanges between the tie plates bent outward while the jaws contracted to but a very small extent. Fracture began at the edge of the outer leg on a line with the rivets near the end of the pin plates. After both angles of this flange had broken, the opposite pair of angles ruptured suddenly through the end rivet of the center batten plate.

No. 6. This was a duplicate of No. 5. Fracture began in one angle through the last rivet in the pin plate while the other angle of the pair broke almost at the same instant through the end rivet of the center batten plate. The opposite pair of angles gave way at about the same time at points diagonally opposite the first two fractures. Jaws at the A end were slightly contracted, while the angles between the batten plates spread outward the same as in Test 5.

No. 7. The four angles broke almost simultaneously through the end rivets in the pin plate, the line of rupture on the web being irregular. The jaws were contracted slightly. There was no other distortion.

No. 8. This was a duplicate of No. 7, with similar fracture. The jaws were not so much contracted, and there was no distortion.

No. 9. Fracture began through the rivet hole at the foot of the second inclined lattice bar, and the adjacent angle gave way before the opposite pair broke on a line of rivets near the end of the pin plate. There was no perceptible deformation of the member except in the very small contraction of the jaws at the B end.

No. 10. This was a duplicate of No. 9. It began to fail through the rivet holes at the end of the pin plate, both angles of one pair breaking simultaneously. The piece then gradually swung around until the opposite angles broke through the rivet holes at the top of the second inclined lattice bar. The center stiffening angle broke on the line of the end rivet at about the same time. There was a very small contraction of the jaws at the B end, but no other deformation except that due to the uneven pull after the first fracture.

Through the courtesy of C. C. Schneider, M. Am. Soc. C. E., and Mr. Frank Heisler, Vice-President of Edge Moor Bridge Works, the author was furnished with comparisons of specimen and full-size tests of eye-bars made of acid and basic open-hearth and Bessemer steels, the average results of 70 tests being scheduled in Table No. 5, which also gives the ratio of the strength of the eye-bars to the strength of the specimens. The specimens were not annealed.

TABLE No. 5.—Comparison of Specimen and Full-Size Eye-Bar Tests.

| Material. | No. of Tests. | Specimens. | | Full Size. | | Percentage of strength developed. |
|---|---|---|---|---|---|---|
| | | E. L. | Ult. | E. L. | Ult. | |
| Basic O.-H..... | 10 | 38 250 | 65 600 | 31 130 | 60 300 | 91.9 |
| Acid O.-H..... | 15 | 36 990 | 65 520 | 30 160 | 57 910 | 88.4 |
| Bessemer ..... | 14 | 37 780 | 60 840 | 32 370 | 55 800 | 91.7 |
| Acid & Basic O.-H. | 31 | 36 350 | 62 370 | 31 420 | 56 970 | 91.3 |
| Average..... | 70 | 37 342 | 63 582 | 31 270 | 57 745 | 90.8 |

By comparing the percentages in Table No. 5 with the results given in Table No. 4, it will be observed that Tests 1, 2, 5 and 6 do not make as good a showing as eye-bars. Tests 7 and 8 are almost as good, while Tests 3, 4, 9 and 10 make even a better showing, so far as the ultimate strength is concerned. All tests in the series, with the exception of Nos. 1 and 2, show a very large elastic limit, while the elongation without exception was decidedly small.

The results of these tests seem to indicate that built-up I sections having double lattice bars for webs, such as Nos. 1 and 2, and those having batten plates, as Nos. 5 and 6, are not so effective as members with single lattice or solid webs. Double lattice drew the flange angles together and caused secondary bending strains which were not relieved even by the eccentric attachment of the pin plates, and the bending had a tendency to cause fracture to start from the edges of the inner legs of the angles at a rather low indicated stress.

Batten plates without lattice did not prevent an outward bending of the angles, and results were not much better than obtained for double lattice.

On the other hand, single lattice, such as in Nos. 3, 4, 9 and 10, and a solid web plate as Nos. 7 and 8, gave the least distortion. There is, therefore, so far as can be discovered from tests of this series, no reason why built-up members of these three types should not be used with the same unit stress as is allowed for eye-bars, and in cases of solid webs no reason why the web plate should not be considered as effective section. This is the practice with some engineers, although others insist upon lower stresses for built members used in tension. The continuation of these tests in series *B* and *C* may temper whatever conclusions one may draw from results of each series considered independently,

consequently series *A*, *B* and *C*, also possibly *D*, should be studied in conjunction with each other, and conclusions not be drawn too hastily.

*Series B, Tests 11 to 18.*—This series was made with the same object in view as outlined for Series *A*, and in addition an effort was made to ascertain whether the connection of both legs of the main angles to pin plates, as in pieces 15 to 18, would be of any material benefit.

There are four designs in pairs, each having two webs and four angles, the pins passing through the webs as shown in Fig. 2. The data in connection with the designs of these pieces are as follows: Gross area, 8.64 sq. ins.; holes deducted, 14, assumed to be $\frac{8}{9}$ in. diameter; net area, 6.72 sq. ins. on zigzag line. The sections through and back of the pin holes are excessive in every case.

The material was soft basic open-hearth steel rolled by the Carnegie Steel Company. Members were made and tested at Edge Moor.

The elastic limit in Table No. 6 was determined by the drop of the beam. The elongation was taken in a length of 8 ins.

TABLE No. 6.—SPECIMEN TESTS, SERIES B.

| | Angles. | Plates. | Average. |
|---|---|---|---|
| Elastic limit | 39 900 lbs. | 36 600 lbs. | 38 250 lbs. |
| Ultimate strength | 57 900 lbs. | 59 300 lbs. | 58 600 lbs. |
| Elongation | 27.00 % | 25.60 % | 26.30 % |
| Reduction | 52.00 % | 57.00 % | 54.50 % |
| Fracture | Close silky. | Close silky. | Close silky. |

The elastic limit of built members as determined by gauge and scaling agreed in all tests in Table No. 7, except Nos. 11, 12 and 14, in which cases scale began to drop at stresses of 45 200, 42 500 and 44 670 lbs. per square inch respectively. Elongation was measured back to back of pin holes the same as before.

TABLE No. 7.—RESULTS OF TESTS IN DETAIL, SERIES B.

| | Elastic Limit. | | Ultimate Strength. | | Elongation. Inches. |
|---|---|---|---|---|---|
| | Total. | Per sq. in. | Total. | Per sq. in. | |
| | Pounds. | Pounds. | Pounds. | Pounds. | |
| 11 | 341 200 | 50 774 | 348 800 | 51 904 | 0.94 |
| 12 | 254 000 | 37 798 | 326 000 | 48 512 | 1.94 |
| 13 | 314 700 | 46 830 | 375 300 | 55 848 | 1.81 |
| 14 | 269 200 | 40 060 | 341 200 | 50 774 | 2.31 |
| 15 | 307 100. | 45 699 | 326 100 | 48 526 | 1.56 |
| 16 | 303 300 | 45 134 | 326 100 | 48 526 | 1 31 |
| 17 | 242 600 | 36 101 | 310 900 | 46 264 | 0.81 |
| 18 | 310 900 | 46 265 | 322 300 | 47 961 | 2.06 |

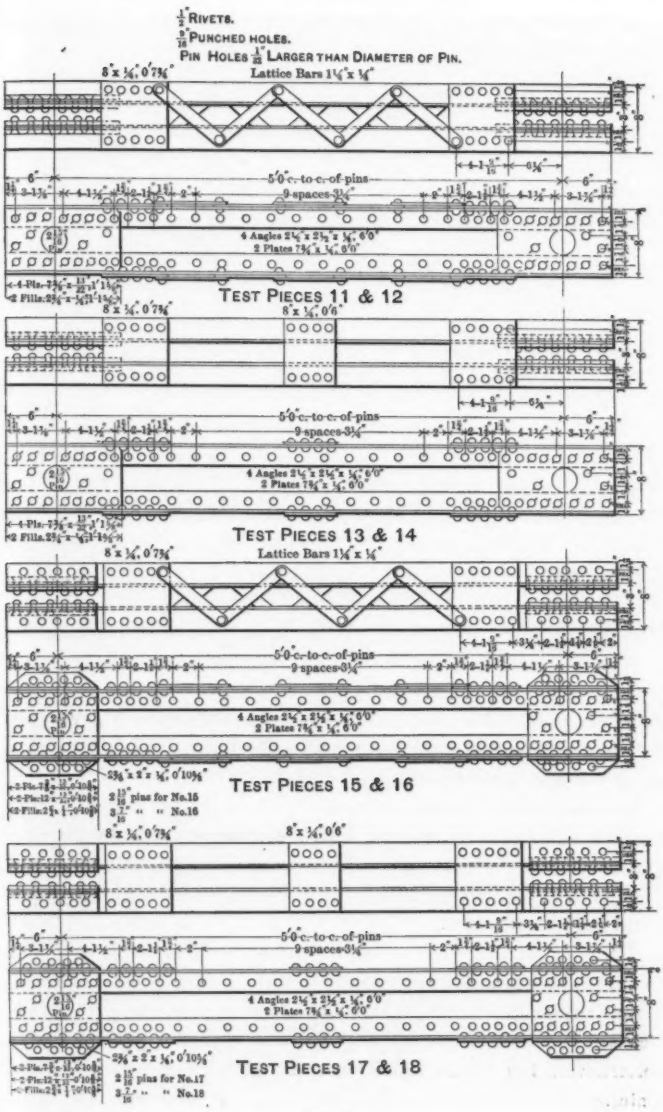
½" Rivets.
⅝" Punched holes.
Pin Holes 1/32" Larger than Diameter of Pin.
8" x ¼", 0' 7¾"
Lattice Bars 1¼" x ¼"
5'0" c.-to-c.-of-pins
9 spaces 3¼"
4 Angles 2½" x 2½" x ¼", 6'0"
2 Plates 7¾" x ¼", 6'0"
Test Pieces 11 & 12
8" x ¼", 0' 6"
Test Pieces 13 & 14
Test Pieces 15 & 16
Test Pieces 17 & 18

Fig. 2.

The average results, together with the comparison of strength of members with specimens, are given in Table No. 8.

TABLE No. 8.—AVERAGE RESULTS AND COMPARISON WITH SPECIMENS.

| | Pounds per Square Inch. | | Comparison with Specimens. | | |
|---|---|---|---|---|---|
| | Elastic limit. | Ultimate strength. | Elastic limit. | Ultimate strength. | Remarks. |
| 11, 12.. | 44 286 | 50 208 | 115.8% | 85.7% | Lattice bars. Narrow pin plates. |
| 13, 14.. | 43 445 | 53 311 | 113.6% | 91.0% | Batten plates. Narrow pin plates. |
| 15, 16.. | 45 417 | 48 526 | 118.7% | 82.8% | Lattice bars. Wide pin plates. |
| 17, 18.. | 41 183 | 47 113 | 107.7% | 80.4% | Batten plates. Wide pin plates. |

See Plate II for the appearance of the members after fracture.

The pins used in Tests 11 to 15 were soft steel and were bent and sheared considerably, the bending amounting to $\frac{1}{16}$ in. in 15 ins. In Tests 16, 17 and 18 the pins were case-hardened, and the two $\frac{15}{16}$-in. pins bent $\frac{3}{16}$ in., while the three $\frac{7}{16}$-in. pins bent about $\frac{1}{16}$-in., without any shearing or measurable distortion of holes.

The fractures, except where noted otherwise, were silky and practically free from granulation. Other observations taken during the tests are as follows:

No. 11. Fracture began in one top angle which broke through the end rivet hole in the batten. The other parts followed quickly, the parts being distorted during the course of rupture. This distortion was not noticeable until the piece began to fail.

No. 12. This was a duplicate of No. 11. Fracture began through the end lattice rivet of one angle, and then continued through the plate and other angle. The first side had broken completely through before the opposite side began to fail. There was no noticeable distortion until failure began.

No. 13. Fracture began through one end rivet in the batten, and continued through both angles and the plate of one segment on a line with the end rivets in the pin plate. Tests were stopped before the adjacent segment failed. There was no observable distortion, but it was noticed that the angles had pulled out $\frac{1}{16}$ in. from under the pin plates.

PLATE II.
TRANS. AM. SOC. CIV. ENGRS.
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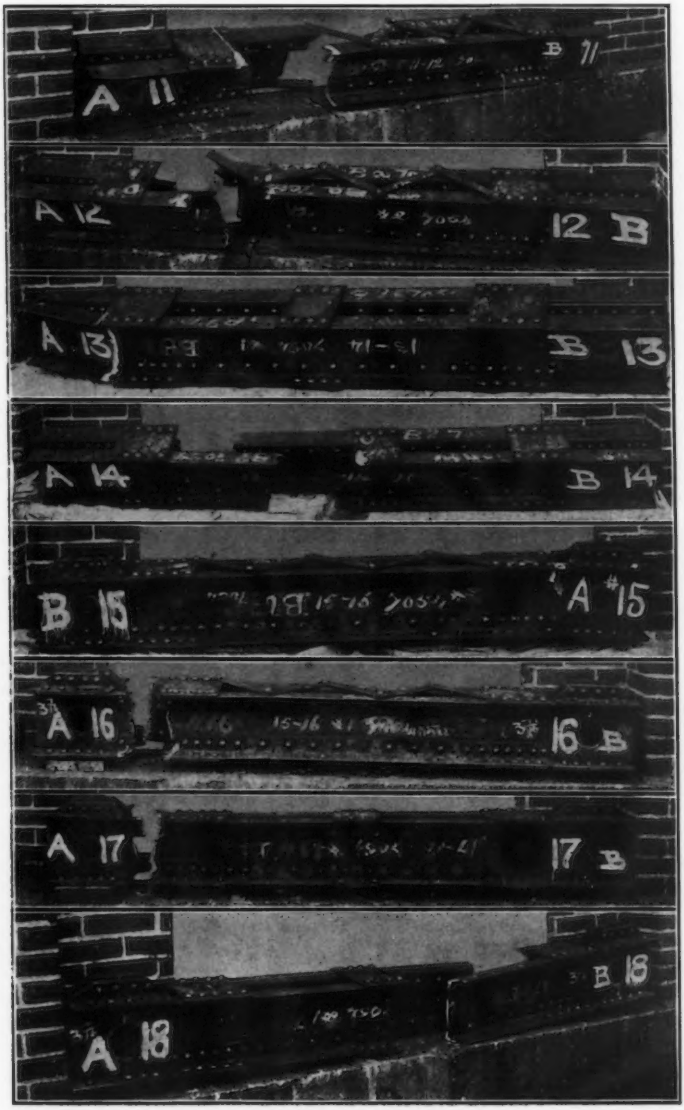
GREINER ON TESTS OF BRIDGE MEMBERS.



No. 14. This was a duplicate of No. 13. The angles again pulled out perceptibly from under the pin plates. Fracture began through the end rivet in the center batten, the top angle first breaking through, then the web, then the bottom angle, and the whole of the other side simultaneously. There was no distortion until after fracture began.

No. 15. Broke at one side only at the end of the batten plate. The pin, which was not hardened, bent $\frac{1}{16}$ in. in a length of 15 ins., and the broken jaw bent outward and slipped off the pin. There was no observable distortion.

No. 16. This was a duplicate of No. 15, except the holes in this member were bored for three $\frac{7}{16}$-in. pins. Fracture started in the end rivet of the pin plate and went almost straight through the line of rivets in the plate. One batten rivet was sheared off. Fracture was coarsely granular in character where failure first began.

No. 17. Broke through the rivets near the end of the pin plates on both sides almost simultaneously. Examination after fracture revealed a crack in one angle across the rivet hole at the end of the end batten plate. Fracture showed considerable granulation.

No. 18. This was a duplicate of No. 17, except the holes were bored for three $\frac{7}{16}$-in. pins. It began to break through the end batten plate rivets on one side. After this side had broken through, the other side held for some time, finally giving way through the line of rivets in the middle batten. There was no distortion except what occurred during rupture.

It was expected that members designed with two web plates and four angles, having pins passing directly through the webs, would give better results than could be obtained from four angles connected together in pairs as in Series *A*. It was also expected that Tests 15 to 18 would be somewhat stronger than Tests 11 to 14, because in the former the pin plates were attached to both legs of the flange angles. These expectations were not realized.

As the pins in these four members had been case-hardened and bent very little during the testing, it is probable that the mere fact that the pin plates did not extend quite up to the first batten plates, will account for members 15 to 18 failing at lower stresses than members 11 to 14, which had longer pin plates to stiffen the jaws, although the members were subjected to the additional strains due to the bending of the soft pins. The attachment of the pin plates to both legs of the

flange angles was evidently of no benefit when taken in conjunction with short pin plates, but had the pin plates been longer, the author believes that the results would have been different.

These tests do not show any advantage of lattice bars over batten plates, members 11 and 12 with lattice giving worse results than similar members, 13 and 14, with battens, while latticed members 15 and 16 were better than similar members, 17 and 18, which had battens.

Members 13 and 14, with battens, are the only ones in the series which developed as high results as eye-bars. Members 11 and 12 (latticed) gave fair results, about as good as 5 and 6 with battens in Series *A*, members 15 and 16 (latticed) about as good as 1 and 2 (double latticed), while 17 and 18 (with battens) developed the least strength of any in the two series.

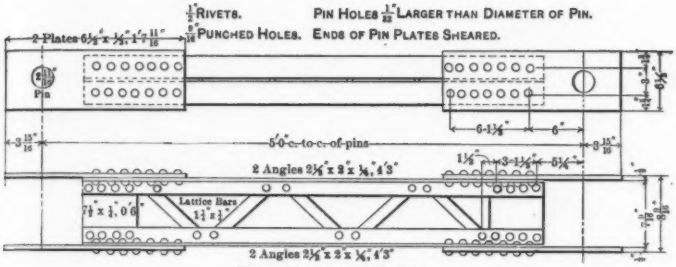
*Series C—Tests 19 to 26.*—The members in this series were designed on the basis of 10 000 lbs., 15 000 lbs., and 20 000 lbs., for shearing, tensile and bearing stresses respectively, little or no excess having been placed in any connection, but the pin plates in Tests 25 and 26 were made weaker on the line of the rivet holes than through the pin holes or the body of member.

The eight members tested covered four different designs, somewhat similar to those in Series *A*, but having the rivets so arranged that but one hole need be deducted from the gross area of each angle. The ends of the pin plates were sheared off (see Fig. 3).

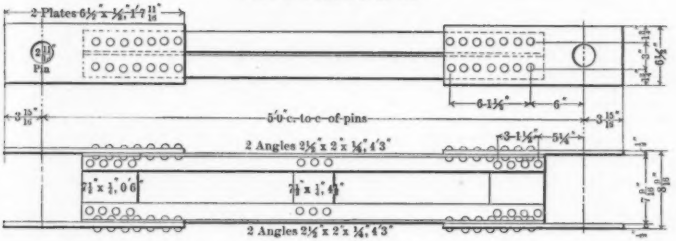
TABLE No. 9.—Data Relating to Members of Series C.

| Members. | Area. | | Holes deducted. | Percentage Increase or Decrease. | | | | |
|---|---|---|---|---|---|---|---|---|
| | Gross. | Net | | Bearing on pins. | Net Area. | | | Rivets. |
| | | | | | Through pin holes. | Back of pin holes. | Front of pin holes. | |
| 19 to 24 .... | .24 | 3.62 | 4 | 0.0 | +4.4 | — 4.63 | +44.75 | +1.1 |
| 25 and 26... | 6.12 | 5.18 | 6 | +3.4 | +2.3 | −11.46 | −21.9 | +2.9 |
| | Pin plate. | 4 25 | | | | | | |

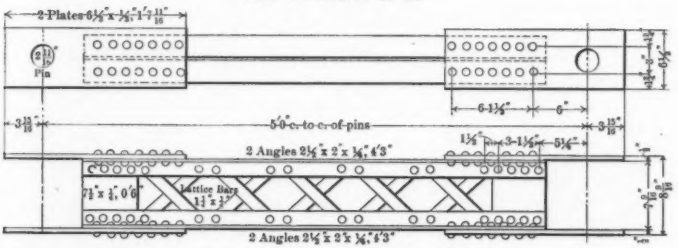
The negative percentage in this table for the area back of the pin holes was based on the assumption that the required area should equal three-fourths of the area across the pin holes, or, in other words, that this area should be proportioned for double shear on the shortest line from the edge of the hole.


7/8" Rivets. Pin Holes 1/32" Larger than Diameter of Pin.
5/16" Punched Holes. Ends of Pin Plates Sheared.
2 Plates 6 1/2" x 1/2" 1'-7 11/16"
Pin
6-1 1/4"
6"
5'-0" c.-to-c.-of-pins
2 Angles 2 1/2" x 2" x 1/4" 4'3"
Lattice Bars
2 Angles 2 1/2" x 2" x 1/4" 4'3"

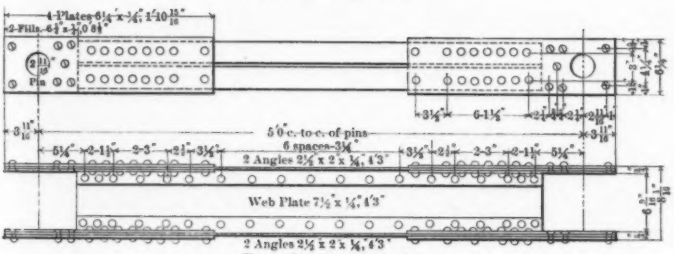
TEST PIECES 19 & 20




TEST PIECES 21 & 22




TEST PIECES 23 & 24


4 Plates 6 1/4" x 1/4" 1'-10 15/16"
2 Fills. 6 1/4" x 1/4" 0 8 1/4"
Pin
5'-0" c.-to-c. of pins
6 spaces 3 1/4"
2 Angles 2 1/2" x 2" x 1/4" 4'3"
Web Plate 7 1/2" x 1/4" 4'3"
2 Angles 2 1/2" x 2" x 1/4" 4'3"

TEST PIECES 25 & 26

FIG. 3.

The material was soft basic open-hearth steel rolled by the Carnegie Steel Company. Members 19 to 22 were manufactured by Youngstown Bridge Company, 23 to 26 at Edge Moor, and all were tested at Edge Moor.

Specimens tested in the same manner as in Series *A* developed the properties given in Table No. 10.

TABLE No. 10.—SPECIMEN TESTS, SERIES C.

| | ANGLES. | PLATES. |
|---|---|---|
| | Average of 4. | Average of 3. |
| Elastic limit | 35 650 lbs. | 38 750 lbs. |
| Ultimate strength | 55 275 lbs. | 56 515 lbs. |
| Elongation | 28.75% | 29.0% |
| Reduction | 56 25% | 58.5% |
| Fracture | Silky. | Silky. |

The gauge determinations of the elastic limit of built members are given in Table No. 11. The limits as indicated by scaling were essentially the same, except in members 20, 22 and 24, where scaling was first noticed at 25 100 lbs., 37 200 lbs., and 34 700 lbs., per square inch respectively. The elongation was measured from back to back of pin holes.

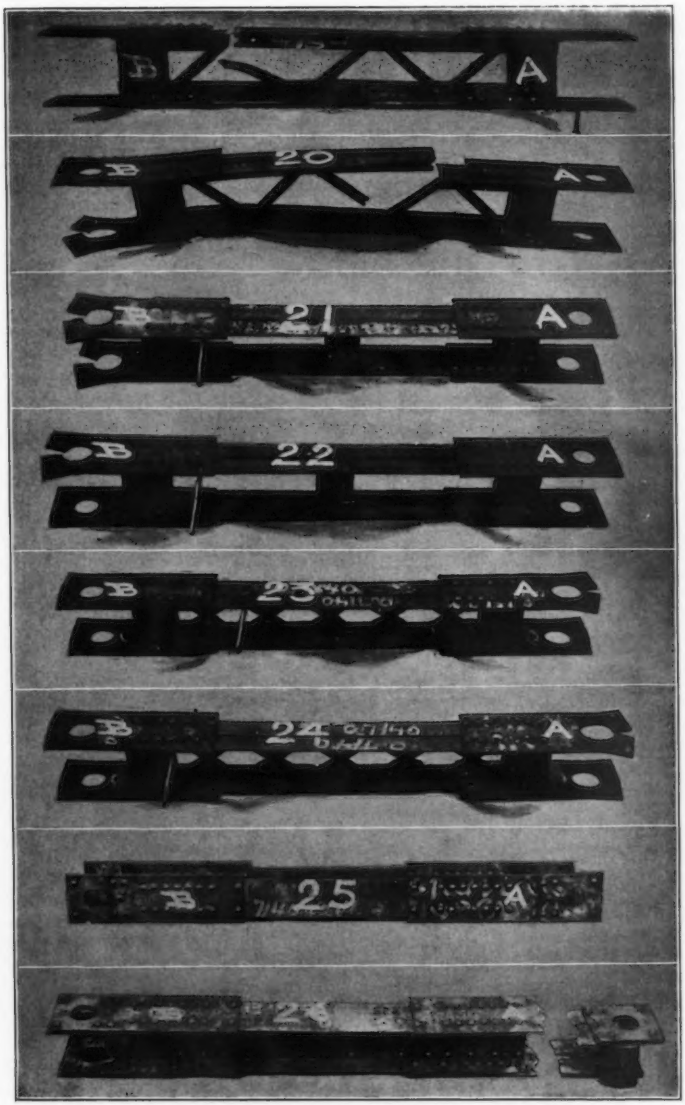
TABLE No. 11.—RESULTS OF TESTS IN DETAIL, SERIES C.

| Member. | ELASTIC LIMIT. | | ULTIMATE STRENGTH. | | ELONGATION. | |
|---|---|---|---|---|---|---|
| | Total. | Per square inch. | Total. | Per square inch. | Total. | Pinholes. |
| | Pounds. | Pounds. | Pounds. | Pounds. | Inches. | Inches. |
| 19 | 166 800 | 46 080 | 174 400 | 48 180 | 1.62 | 0.25 |
| 20 | 106 000 | 29·280 | 178 200 | 49 230 | Not taken. | 0 25 |
| 21 | 151 600 | 41 885 | 182 000 | 50 280 | " " | 0 25 |
| 22 | 151 500 | 41 860 | 182 000 | 50 280 | 1.69 | 0.25 |
| 23 | 155 400 | 42 930 | 174 400 | 48 180 | 2.81 | 0.87 |
| 24 | 151 600 | 41 880 | 166 800 | 46 080 | 2 69 | 0.75 |
| 25 | 182 000 | *42 830 | 212 300 | *49 960 | 2 12 | 0.56 |
| 26 | 189 500 | *44 590 | 219 900 | *51 740 | Not taken. | 0.44 |

* The strain per square inch of members 25 and 26 was taken on a section through the weakest part of the pin plate.

The average results of tests, together with comparisons with specimens, are given in Table No. 12. This table, however, is to be studied cautiously, inasmuch as Nos. 19 and 20 are the only mem-

PLATE III.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXVIII, No. 810.
GREINER ON TESTS OF BRIDGE MEMBERS.



bers which broke through the body, the others having given way back of the pin holes or through the pin plates in front of the pins. The percentages of developed strength are based on stresses on the net section through the body, except in Nos. 25 and 26, where the net section is the least through the rivets in front of the pin holes.

TABLE No. 12.—AVERAGE RESULTS AND COMPARISON WITH SPECIMENS.

| | Pounds per Square Inch. | | Comparison with Specimens. | | Remarks. |
|---|---|---|---|---|---|
| | Elastic limit. | Ultimate strength. | Elastic limit. | Ultimate strength. | |
| 19 and 20..... | 37 680 | 48 705 | 105.7% | 88.1% | Single lattice. |
| 21 and 22..... | 41 872 | 50 280 | 117 5% | 91 0% | Battens. |
| 23 and 24..... | 42 405 | 47 130 | 119.0% | 85 3% | Double lattice. |
| 25 and 26..... | 43 710 | 50 850 | 112.8% | 90.0% | Web. |

The pins used were case hardened, and bent so slightly that it was necessary to change but once during the testing.

The illustrations on Plate III should be compared with the following notes of observations taken during the progress of the tests:

No. 19.—First rupture occurred at end B, both angles breaking through the second lattice rivet, one rather suddenly, the other with slightly more tenacity. A crack also appeared in the same angles through the end rivets of the pin plate at the same end. After breaking through on this side, the member swung slowly round, and the other two angles broke through the third lattice rivet from the B end, tearing rather than snapping. On one side of the member the fracture of the upper angle was silky and of the lower wholly crystalline, while on the opposite side these conditions were reversed. There was very small distortion in any part of the member, except what occurred after one side had broken.

No. 20.—This was a duplicate of No. 19. The rupture in this member began at end A, at the same point as in the previous case, both angles failing at the same time. The member then swung around, splitting the middle lattice bar away from its rivet and failing at end B by splitting the pin plate back of the pin hole. The fracture through the angles at end A was wholly silky, while the crack in the pin plate had a fibrous appearance. One pin plate at end A was drawn in about $\frac{1}{4}$ in. around the pin hole, with a perceptible bulging

out at the end, with several cracks starting. The distortion shown in the illustration took place after the angles fractured.

No. 21.—Batten plates. This member broke out back of the pin plates in end B, both pin plates failing simultaneously, one with a coarse granular or crystalline, and one with a fibrous, fracture. The deformation of the member in this case was considerable, and it was drawn in about $\frac{1}{4}$ in. near the middle batten, or rather it was bulged out this amount between the battens. Both pin plates at end A started to crack in the end.

No. 22.—This was a duplicate of No. 21. It broke suddenly through one pin plate at end B, with crystalline fracture. There were cracks in the end of both pin plates at end A. The member was slightly bent, but not to the extent observed in No. 21.

No. 23.—Double lattice. This broke suddenly through one pin plate at end A, the other pin hole at this end elongating about $\frac{7}{8}$ in. At end B both pin holes were elongated $\frac{3}{4}$ in., and the plates showed cracks in the ends. There was a perceptible reduction of section at the pin holes. The member was considerably distorted about the jaws, and the two flanges were drawn together perceptibly by the pull of the lattice.

No. 24. This was a duplicate of No. 23. One pin plate at end A broke out to the end, the other pin hole elongating $\frac{3}{4}$ in. The fracture had a fibrous appearance. At end B both pin holes were elongated $\frac{9}{16}$ in. and showed cracks in the ends of the plates. There was considerable contraction of area through the pin holes and distortion due to the pull of the lattice bars, the same as noted in No. 23.

No. 25. Web plate. The pin plates on one side at end A curled over, the part back of the pin convexing outward and the pin hole elongating $\frac{13}{16}$ in. All three plates at this point were cracked at the ends, but did not break out. The pin plates on the opposite side at this end broke off suddenly with a fine silky fracture, the outside one through the three rivets at the end of the filler farthest from the pin, the inside one through the first row beyond. The reduction of section through both points of fracture was marked, both at this end and the corresponding points at end B. All the pin plates at the B end were cracked in the end.

No. 26. This was a duplicate of No. 25. The pin plates at both sides at end A broke simultaneously in the same manner as in Test 25,

with the same conditions of cracks and reduction at end B. The fracture was fine silky.

No rivets were sheared off in any of the tests, nor were any found loose enough to be detected with the fingers. There was, however, a very general pulling away from the rivets noticed, both in the pin plates and in the lacing, arising no doubt partially from a shearing action and partially from an elongation of the holes.

It is to be noticed that in this series the members having batten plates were better than those having single lattice, just the reverse of results in Series *A*. The developed strength was 91%, which is about equal to the strength of eye-bars and considerably in excess of similar members numbered 5 and 6, which developed but 85.06 per cent.

The strength of the members numbered 19 and 20 (single lattice) was lower than that of eye-bars and much lower than similar members numbered 3 and 4.

The author cannot account for these results unless they are due to the hardened pins used in Series *C*, which may possibly have given more direct stress than was obtained from the soft pins in Series *A*, in which case it would appear that built I members with battens are just as effective as similar members with lattice bars.

Members numbered 23 and 24 show up better than similar members numbered 1 and 2, although the pin plates did not develop the full strength through the body of the member. Members numbered 25 and 26 (web plates) failed in the weakest part of the connection, and the tests show nothing except that connection plates of this size, strained in this manner, gave results 10% less than given by specimens.

The tests in Series *C* seem to show that pin plates having sheared ends should have a net area back of the pin holes of not less than 75% of the net area through the pin holes. They also apparently sustain the assumed relative values of 10 000, 15 000 and 20 000 lbs. respectively for shearing, tensile and bearing stresses.

*Résumé of Series A, B and C.*—The natural query upon the completion of a number of tests is: what are the conclusions? Conclusions, however, are indicated rather than proved in most cases pertaining to limited investigations concerning structural material. The following, therefore, are no exceptions to the general rule, and are merely those which appear to the author to be indicated by the results

of the 26 tests in Series *A*, *B* and *C*. All facts in connection therewith are given in the paper. The indicated conclusions are as follows:

*First.*—Built I-shaped tension members composed of four angles connected in pairs by single lattice bars, or by batten plates spaced at short intervals, and having the end connection plates riveted to the projecting legs of the flange angles, will develop an ultimate strength which, when compared with tests of specimens cut from the member, will be proportionately as high as the ultimate strength of eye-bars compared with their specimens, when the eye-bars are annealed and the specimens not annealed.

*Second.*—Similar members with solid webs instead of lattice or battens are fully as strong as eye-bars per unit of section, and the full net area of web plates can be taken with the flange angles as available section, provided the area of web plates does not exceed one-half the area of the four angles, and the width of the plate in the clear of the flange angles does not exceed the width covered by the legs of the flange angles.

*Third.*—Similar members with double-latticed webs are probably inferior to any of the above, and their ultimate strength is proportionately less than the ultimate strength of eye-bars.

*Fourth.*—Similar members with open webs having lattice bars arranged alternately perpendicular and inclined to the long axis of the member, will show the least distortion when pulled to destruction, and will give the highest ultimate strength.

*Fifth.*—Box-shaped sections composed of two web plates and four angles, connected top and bottom by lattice or battens, and having pin connections through the web plates are, as tension members, no better than, if as good as, built I sections.

*Sixth.*—Segments of box-shaped members when connected by single lattice or batten plates, placed at short intervals, show no particular advantage of the lattice bars over the batten plates.

*Seventh.*—Nothing can be gained in box-shaped tension members by connecting the pin plates to both the vertical and horizontal legs of the flange angles, when the pin plates do not extend beyond the end rivets in the end batten plates.

*Eighth.*—The relative values of 10 000 lbs., 15 000 lbs., and 20 000 lbs., respectively, for shearing, tensile and bearing stresses are about correct, but there is an indication that the bearing values should be somewhat less.

*Series D, Tests 27 to 42.*—Sixteen test pieces, numbered from 27 to 42 in Fig. 4, were designed so that the area of the section back of the pin holes varied from about 32% to 96% of that across the holes. The tests are divided into four groups, A, B, C and D, there being four tests to each group, two with sheared ends and two (designated by letter M)

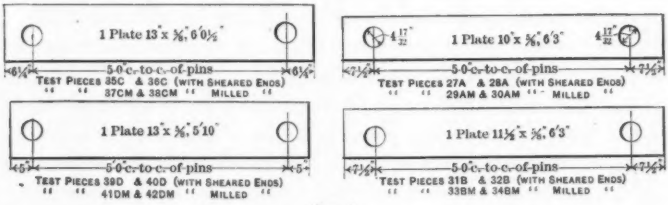

1 Plate 13" x ⅜", 6'0½"
6¼"
5'0" c.-to-c. of pins
6¼"
Test Pieces 35C & 36C (with Sheared Ends)
" " 37CM & 38CM " Milled "
1 Plate 10" x ⅜", 6'3"
4 17/32"
4 17/32"
7½"
5'0" c.-to-c. of pins
7½"
Test Pieces 27A & 28A (with Sheared Ends)
" " 29AM & 30AM " Milled "
1 Plate 13" x ⅜", 5'10"
5"
5'0" c.-to-c. of pins
5"
Test Pieces 39D & 40D (with Sheared Ends)
" " 41DM & 42DM " Milled "
1 Plate 11½" x ⅜", 6'3"
7½"
5'0" c.-to-c. of pins
7½"
Test Pieces 31B & 32B (with Sheared Ends)
" " 33BM & 34BM " Milled "

FIG. 4.

with milled ends. The material was soft open-hearth steel made and tested at Pencoyd. The object of the tests was to ascertain the proper amount of material required back of the holes in pin plates, and also what benefit can be derived from milling the ends.

Specimens cut from the various plates developed the properties given in Table No. 13.

TABLE No. 13.—SPECIMEN TESTS, SERIES D.

| Size. | 10 × ⅞ in. | 11½ × ⅞ in. | 13 × ⅞ in. |
|---|---|---|---|
| Elastic limit | 42 500 lbs. | 34 100 lbs. | 35 500 lbs. |
| Ultimate strength | 60 300 lbs. | 54 700 lbs. | 58 700 lbs. |
| Elongation | 38.25% | 34.00% | 28.25% |
| Reduction | 62.00% | 64.00% | 51.80% |
| Fracture | S. cup. | S. ½ cup. | S. ang. |

In Table No. 14 will be found all data and details of the results of the tests, the ratio in the fourth column being the ratio of the net area back of the pin holes to that across pin holes. Attention is called to the comparatively high ultimate strength given for Test 28 A. In view of the fact that it is so much greater than the three others of the same group, and that there is no apparent cause for this, the author is inclined to question the accuracy of the reading, and has therefore not considered this test in the comparative result given in Table No. 15.

TABLE No. 14.—Results of Tests in Detail, Series D.

| Member. | Net Section. | | Ratio. | Total. | | Per Square Inch. | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Elastic limit. | | Ultimate strength. | |
| | Across hole. | Back of hole. | Back / Across. | Elastic limit. | Ultimate strength. | Through pin. | Back of pin. | Through pin. | Back of pin. |
| | Sq. Ins. | Sq. Ins. | | lbs. | lbs. | lbs. | lbs. | lbs. | lbs. |
| 27 A | 3.42 | 3.27 | .96 | 96 000 | 165 000 | 28 100 | 29 300 | 48 300 | 50 400 |
| 28 A | 3.42 | 3.27 | .96 | 101 000 | 201 600 | 29 500 | 30 900 | 59 000 | 61 600 |
| 29 AM | 3.42 | 3.27 | .96 | 98 000 | 162 000 | 28 700 | 30 000 | 47 400 | 49 500 |
| 30 AM | 3.42 | 3.27 | .96 | 100 000 | 163 000 | 29 300 | 30 600 | 47 700 | 49 800 |
| 31 B | 4.36 | 3.27 | .75 | 74 000 | 177 000 | 17 000 | 22 600 | 40 600 | 54 100 |
| 32 B | 4.36 | 3.27 | .75 | 54 000 | 185 000 | 12 400 | 16 500 | 42 500 | 56 500 |
| 33 BM | 4.36 | 3.27 | .75 | 90 000 | 170 000 | 20 700 | 27 500 | 39 000 | 52 000 |
| 34 BM | 4.36 | 3.27 | .75 | 54 000 | 177 000 | 12 400 | 16 500 | 40 600 | 54 100 |
| 35 C | 5.29 | 2.49 | .47 | 92 400 | 178 000 | 17 500 | 37 100 | 33 600 | 71 500 |
| 36 C | 5.29 | 2.49 | .47 | 110 000 | 180 000 | 20 800 | 44 200 | 34 000 | 72 300 |
| 37 CM | 5.29 | 2.49 | .47 | 85 000 | 225 000 | 16 000 | 34 100 | 42 500 | 90 400 |
| 38 CM | 5.29 | 2.49 | .47 | 77 000 | 225 000 | 14 600 | 30 900 | 42 500 | 90 400 |
| 39 D | 5.29 | 1.71 | .32 | 120 000 | 141 000 | 22 700 | 70 200 | 26 600 | 82 500 |
| 40 D | 5.29 | 1.71 | .32 | 82 000 | 141 000 | 15 500 | 48 000 | 26 600 | 82 500 |
| 41 DM | 5.29 | 1.71 | .32 | 54 000 | 193 000 | 10 200 | 31 600 | 36 500 | 112 900 |
| 42 DM | 5.29 | 1.71 | .32 | 57 000 | 195 000 | 10 800 | 33 400 | 36 800 | 114 100 |

Table No. 15 gives the average results, comparisons with specimens cut from the plates, comparisons of results of milled and sheared ends, and the ratio of the width to the thickness of plates for all tests in Series *D*, also for the four plates which broke back of pin holes in the tests of Series *C*. The latter are given for convenient study in this connection.

TABLE No. 15.—Average Results and Comparisons.

| Members. | Ratios. | | Pounds per Square Inch. | | Ratios. | |
|---|---|---|---|---|---|---|
| | Back of P. H. / Thickness. | Back of P. H. / Across P. H. | Specimen. | Developed. | Developed. / Specimen. | Milled. / Sheared. |
| A (one test) | 8.4 | .96 | 60 300 | 48 300 | .80 | |
| AM | 8.4 | .96 | 60 300 | 47 550 | .79 | .98 |
| B | 8.4 | .75 | 54 700 | 41 550 | .76 | |
| BM | 8.4 | .75 | 54 700 | 39 800 | .73 | .96 |
| C | 6.4 | .47 | 58 700 | 33 800 | .58 | |
| CM | 6.4 | .47 | 58 700 | 42 500 | .72 | 1.26 |
| D | 4.4 | .32 | 58 700 | 26 600 | .45 | |
| DM | 4.4 | .32 | 58 700 | 36 650 | .62 | 1.38 |
| Series C | 5.2 | .68 | 55 840 | 46 140 | .83 | |

The test pieces were placed in the machine in such a manner that at the top of the plate there was no support back of the pin, while at the bottom there was more or less support given by the jaws which took hold of the pin on each side of the plate. Had the tops been held instead of being absolutely free, the results would in all probability

have been different, and would have furnished the information sought. As the results stand, however, the proper ratio of the area back of holes to that across holes is not definitely indicated, and owing to the buckling of the unsupported metal back of the holes in nine test pieces, the benefit of milled ends is not apparent except for those members which did not buckle.

In actual practice, the ends of the pin plates are not usually supported by adjacent members, and these tests therefore are in accord. Similar results would probably follow the testing to destruction of many tension members, now parts of bridges in service.

Buckling back of upper pin holes occurred in all members in Groups A and B and in but one member of Group C. Three members of Group C and all of Group D broke out in center line of pin without appreciable buckling.

A study of results as given in Tables Nos. 14 and 15 will show that the following points have been brought out by the tests:

*First.*—Pin plates in which the area back of the holes was greater than 75% of the area through the holes, buckled and curled at the unsupported end before breaking out, and the milled ends were no stronger than the sheared ends. Those having a ratio less than 75% broke out without buckling, and the milled ends developed largely increased strength.

*Second.*—The developed stress, per square inch of area, back of the holes varied inversely, while the stress per square inch across the holes varied directly as the amount of metal back of the holes.

*Third.*—Buckling back of the holes took place in all members in which the ratio of length back of holes to thickness was greater than 6.4.

It is evident that had the plates been thicker, there would have been less tendency to buckle. The proper amount of material back of the pin holes, therefore, is not merely a function of the area across the holes, but is some function of the area across the holes, the thickness of the plate, and the diameter of the pins, and cannot be definitely determined from the results of the tests made.

*Series E, Tests 43 to 50.*—The eight tests, in this series, were made for the purpose of comparing the strength of angles of the same dimensions, some having both legs and others but one leg riveted to gusset plates, the ends of the gussets being clamped in the testing machine so as to give, as nearly as possible, conditions similar to those in actual practice (see Fig. 5).

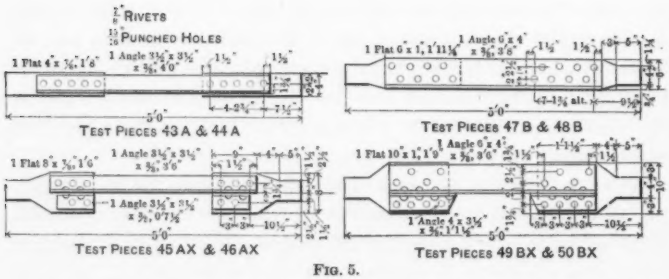



FIG. 5.

The material used was soft open-hearth steel rolled at Pencoyd, at which place all tests were made.

Specimens cut from angles had the properties shown in Table No. 16, which gives the average of two tests cut from each size of angle.

TABLE No. 16.—SPECIMEN TESTS, SERIES E.

| | $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ Ins. | $6 \times 4 \times \frac{3}{8}$ Ins. |
|---|---|---|
| Elastic limit | 38 850 lbs. | 36 300 lbs. |
| Ultimate strength | 59 350 lbs. | 55 200 lbs. |
| Elongation | 28.88% | 33 38% |
| Reduction | 56.85% | 64.80% |
| Fracture | Silky. | Silky. |

Results of the tests of these members are given in Table No. 17, the elastic limit having been determined by the dropping of scale, instead of by the dropping of the beam, and the unit stresses are based on the least net area, whether it be on a straight or broken line. Members marked A represent $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$-in. angles, those marked B $6 \times 4 \times \frac{3}{8}$-in. angles, while the letter X denotes that both legs are connected to gussets, and its omission that but one leg is connected.

TABLE No. 17.—RESULTS, IN DETAIL, OF TESTS, SERIES E.

| No. | Area. | | Elastic Limit. | | Ultimate Strength. | |
|---|---|---|---|---|---|---|
| | Gross. | Net. | Total. | Per sq. in. | Total. | Per sq. in. |
| | Sq. Ins. | Sq. Ins. | Pounds. | Pounds. | Pounds. | Pounds. |
| 43 A | 2.51 | 2.14 | 45 000 | 21 028 | 99 000 | 46 261 |
| 44 A | 2.51 | 2.14 | 57 000 | 26 635 | 91 000 | 42 523 |
| 45 AX | 2 51 | 1.99 | 56 000 | 28 141 | 105 300 | 52 914 |
| 46 AX | 2.51 | 1.99 | 50 000 | 25 125 | 102 700 | 51 608 |
| 47 B | 3.60 | 3.06 | 76 000 | 24 837 | 128 000 | 41 830 |
| 48 B | 3.60 | 3.06 | 72 000 | 23 529 | 129 300 | 42 255 |
| 49 BX | 3.60 | 3.08 | 94 000 | 30 519 | 153 000 | 49 675 |
| 50 BX | 3.60 | 3.08 | 80 000 | 25 974 | 154 000 | 50 000 |

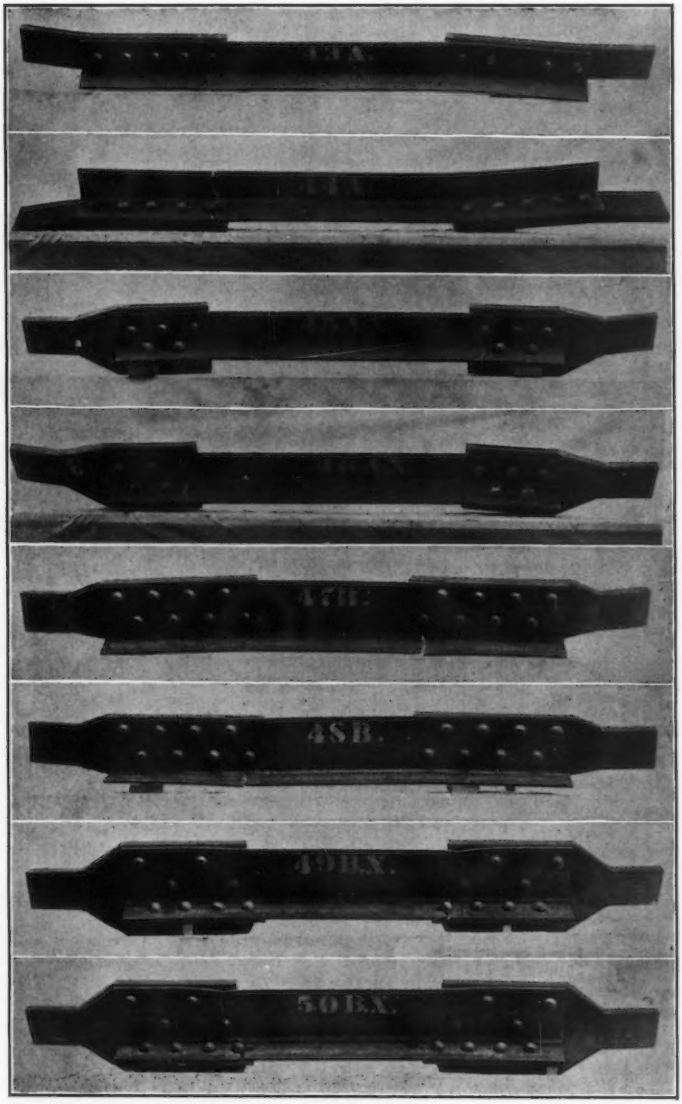

47A.
47B.
48B.
49BX.
50BX.

Table No. 18 gives the average results and comparisons of the two styles of connections, based on the total elastic limit and the total ultimate.

TABLE No. 18.--Comparisons, Series E.

| Sizes. | Legs connected. | Elastic Limit. | | Ultimate Strength. | |
|---|---|---|---|---|---|
| | | Total. | Increase. | Total. | Increase. |
| Inches. | | Pounds. | Percentage. | Pounds. | Percentage. |
| 3½ x 3½ x 5/8 | One. | 51 000 | ..... | 95 000 | |
| 3½ x 3½ x 5/8 | Two. | 53 000 | 3.92 | 104 000 | 9.5 |
| 6 x 4 x 5/8 | One. | 74 000 | ..... | 128 650 | |
| 6 x 4 x 5/8 | Two. | 87 000 | 17.57 | 153 500 | 19.3 |

The average elastic limit and ultimate strength per square inch of least net section, also the percentage comparisons with specimen tests are given in Table No. 19.

TABLE No. 19.—Average Results, Compared with Specimens.

| Tests. | Sizes. | Legs Connected. | Pounds per Square Inch. | | Comparison with Specimens. | |
|---|---|---|---|---|---|---|
| | | | Elastic limit. | Ultimate strength | Elastic limit. | Ultimate strength. |
| | Inches. | | | | | |
| 43, 44..... | 3½ x 3½ x 5/8 | One. | 22 832 | 44 392 | 61.3% | 74.8% |
| 45, 46 ..... | 3½ x 3½ x 5/8 | Two. | 26 633 | 52 261 | 68.6% | 88.1% |
| 47, 48..... | 6 x 4 x 5/8 | One. | 24 183 | 42 042 | 66.6% | 76.2% |
| 49, 50 .... | 6 x 4 x 5/8 | Two. | 28 247 | 49 838 | 77 8% | 90.3% |

The appearance of the different angles after fracture is shown on Plate IV. Pieces 45 and 46 broke square across through one rivet hole, the rivets being staggered 1½ ins.; 47 and 48 broke diagonally through two holes which were staggered 1½ ins., and 49 and 50 broke diagonally through two holes which were staggered 1½ ins. All rivets except those at ends of connection plates were tight after fracture. The rivets at the ends of the connection plates gave evidence of shearing.

By referring to Table No. 18 it will be seen that when both legs are connected there is a gain in ultimate strength of about 9.5% for 3½ x 3½-in. angles, and about 19.3% for the 6 x 4-in. angles; also that there is a gain in the elastic limit.

The single angles developed a strength of from 74.8% to 76.2% of the ultimate strength of the specimens, which is considerably below the average developed strength of eye-bars, but the double connection gave results practically as high as those obtained with bars.

There is no question as to the superiority of the double connection, and it should be used whenever practicable and when conditions require rigidity. On the other hand, the single connection is not so weak as is usually supposed, the developed strength being, so far as these tests go, about 83% of the developed strength of eye-bars, as can be seen by comparing the percentage of developed strength of the angles given in Table No. 19, with the average developed strength of the bars given in Table No. 5.

*Series F.*—This series of tests was made for the purpose of ascertaining to what extent a good structural steel will be damaged, if a part is heated and bent while the other part remains cold. Will an area of weakness develop somewhere between the heated and unheated portions? Will it become brittle or unreliable? Is it absolutely necessary in all cases to anneal the entire piece?

The fact that steel eye-bars, when not annealed, will sometimes break through or near the neck, and when partly annealed, through a section somewhere near the black heat area, led to the general practice of requiring that all members which have been partly heated must be annealed. There is no question concerning the beneficial effects of annealing when done properly, but cases sometimes arise when annealing in a furnace becomes impracticable, and the member must be used without any annealing, or resort must be had to the questionable wood fire. The author was recently called upon to decide such a case, and stipulated that if a series of tests indicated that an angle bent partly hot and partly cold did not become brittle, or was not injured to a greater extent than when the entire angle was bent cold, he would waive annealing for the members in question. The six groups of tests given in Table No. 20 were then made, and as the results did not indicate any marked superiority of the cold-worked angles over those which had one end worked hot, and as no brittleness or particularly weak area was found to exist in the steel tested, the manufacturer was allowed to heat and bend his angles without annealing. The author does not wish to be misunderstood as to his stand in this particular case. The angles in question were not to be used in a

railroad bridge, but in a roof truss subjected to but little vibration and no shock. If cold-bent angles were good enough for this case, then those bent hot and not annealed were no worse.

TABLE No. 20.—Specimens Cut from Unworked Material.

| Cut from. | Physical Properties. | | | Chemical Properties. | | | |
|---|---|---|---|---|---|---|---|
| | Elas. lim. | Ult. str. | Elong. | Carbon. | Phos. | Mn. | S. |
| | Pounds. | Pounds. | Per Cent. | | | | |
| 6 x 3½ x ⅜-in. angles.. | 36 000 | 62 100 | 30.0 | 0.17 | 0.015 | 0.40 | 0.038 |
| 5 x 3 x ⅜-in. angles.... | 38 700 | 66 700 | 28.75 | 0.24 | 0.016 | 0.45 | 0.040 |
| 3½ x 2½ x ¼-in. angles | 39 800 | 60 600 | 28.75 | 0.19 | 0.017 | 0.45 | 0.040 |
| 3 x 2 x ¼-in. angles.... | 39 000 | 61 200 | 27.5 | 0.24 | 0.021 | 0.45 | 0.038 |
| 3 x ½-in. flats..... | 39 500 | 62 000 | 25.0 | 0.22 | 0.010 | 0.47 | 0.040 |
| 1-in. square rods..... | 36 000 | 60 300 | 31.0 | 0.21 | 0.015 | 0.46 | 0.039 |

The tests were divided into six groups, and, except where otherwise noted, were bent in a gag press to a circular arc, the ordinate of which was measured to a 2-ft. chord. The sizes of the test pieces are shown in Fig. 6.

Punching was not regular, the holes varying in number from 5 to 7, and in center to center distance from 3 to 6 ins.

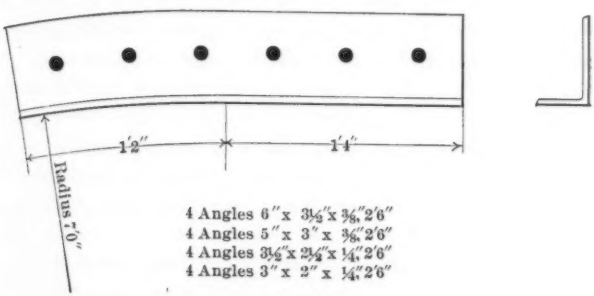



Fig. 6.

Group 1.—Four angles 2 ft. 6 ins. long, one of each size, were prepared by heating one-half length, bending, and then punching after the pieces became cold.

The 6 x 3½ x ⅜-in. angle bent 3½ ins. without developing cracks. It was then hammered out flat and cracked along the root about 6 ins. on each side of the center. A short crack was produced across the root

about 8 ins. from the cold end. There were slight cracks in the rivet holes.

The 5 x 3 x $\frac{3}{8}$-in. angle bent about $3\frac{1}{2}$ ins., without injury. It was then hammered out flat and cracked for a distance of 3 ins. across the root at the center of the piece.

The $3\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$-in. angle bent $3\frac{5}{8}$ ins. before a crack started from the hole near the center of the piece. The angle was broken by continued gagging. The fracture was silky, with no granulation.

The 3 x 2 x $\frac{1}{4}$-in. angle bent 4 ins., when two cracks started at holes $8\frac{1}{2}$ and $11\frac{1}{2}$ ins. from the cold end. The angle was broken by continued gagging. The fracture was granular from the hole to the root of the angle.

Group 2.—Four angles as in Fig. 6, bent and punched cold.

The 6 x $3\frac{1}{2}$ x $\frac{3}{8}$-in. angle was not bent by gagging. While being hammered out flat, it cracked along the root.

The 5 x 3 x $\frac{3}{8}$-in. angle was not bent by gagging. While being hammered out flat it cracked along root.

The $3\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$-in. angle bent $3\frac{1}{2}$ ins., and cracks started in holes 12 and 15 ins. from the straight end.

The 3 x 2 x $\frac{1}{4}$-in. angle bent $3\frac{1}{4}$ ins., and a crack started in a hole 15 ins. from the straight end.

Group 3.—Four angles as in Group 1, one-half length heated and bent. They were punched after the pieces had been annealed.

The 6 x $3\frac{1}{2}$ x $\frac{3}{8}$-in. angle bent 3 ins., and a crack started in one of the rivet holes.

The 5 x 3 x $\frac{3}{8}$-in. angle bent $3\frac{7}{8}$ ins., without cracking.

The $3\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$-in. angle bent $3\frac{3}{4}$ ins., without cracking.

The 3 x 2 x $\frac{1}{4}$-in. angle bent 2 ins., and cracked through a hole 8 ins. from the unheated end.

Group 4.—Four angles as in Group 1, bent and punched cold and then annealed.

The 6 x $3\frac{1}{2}$ x $\frac{3}{8}$-in. and 5 x 3 x $\frac{3}{8}$-in. angles bent $2\frac{1}{2}$ ins., without cracking.

The $3\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$-in. angle bent $3\frac{3}{4}$ ins., without cracking.

The 3 x 2 x $\frac{1}{4}$-in. angle bent $4\frac{3}{8}$ ins., without cracking.

Group 5.—Two rods 1 in. square by 2 ft. 6 ins. long were heated for one-half their length and cooled in the open air. These rods were each placed in a testing machine and were struck several sharp blows

with a hammer while under loads of 25 000, 30 000 and 35 000 lbs. Both broke in the middle at a stress of 58 700 lbs. and a reduction of 50 per cent. The fracture was silky.

Group 6.—Four flats 3 ins. by $\frac{1}{2}$ in. by 2 ft. 6 ins. were heated for one-half their length and cooled in the open air. Two of them were bent at the center to 180° and closed down with a slight crack opening in one. One flat was nicked at the heated and unheated ends and at the middle. The ends were bent 180° to a 1-in. circle, and the middle 180° to a 3-in. circle, before breaking. The fracture in all cases was fine and silky.

The fourth flat was nicked in the middle, bent 180° and flattened down without fracture.

From these tests the benefits of annealing in Group 4 are apparent, as no cracks whatever developed, although the members were bent as much as possible by the gag. The two larger angles twisted to such an extent that the bending could not be carried to the same point as in the other groups.

There was no apparent benefit derived by annealing and then punching, as in Group 3, and Group 2 was not superior to Group 1.

The impact tests in Group 5 and bending tests in Group 6 failed to reveal any injury due to partly heating steel of this character, except that the ultimate strength was reduced, which, however, does not indicate anything, except that the piece had been softened the same as it would have been by annealing.

*Conclusion.*—In conclusion, the author wishes to state that while the foregoing six series of tests are incomplete and do not prove anything definite, they have at least developed suggestions which may be regarded as useful information, and which should encourage further investigation by those who have the opportunity. The author also begs to acknowledge the courtesies extended by the bridge companies which furnished the facilities for making the tests, and the thoroughness with which Mr. W. R. Edwards, his assistant, carried out instructions while the testing was in progress.

## DISCUSSION.

Mr. Russell. S. BENT RUSSELL, M. Am. Soc. C. E.—Referring to Table No. 3 it will be seen that the greatest elongation shown by any one of the ten bars of the series was 2.41 ins., about 4.3% of the original length. The least elongation shown was something less than 2%, and the average for the ten was about 2.8 per cent. It is of interest to compare these results with those obtained with eye-bars.

A great many tests of full-size steel eye-bars have been made and published. A study of them will show that a steel eye-bar will stretch from 10 to 22% of its original length before rupture takes place. It may be safely said that good steel eye-bars ought to stretch 13% on the average. Now, it is evident that a properly made eye-bar of considerable length will show the true proportionate elongation for a tension bar of uniform cross-section. Thus the maximum possible elongation for a tension member is obtained in the eye-bar.

Returning to the author's bridge members, it will be found that the average bar gave 2.8 and the best bar 4.3 out of a possible 13. May it not be concluded, then, from this comparison that a built-up tension bridge member will show a proportionate elongation which will be less than one-fourth of that given by a solid tension member?

Now, it is generally recognized that ductility, or the quality which makes great elongation possible, is of high importance in bridge material. No bridge steel is accepted unless it will show great elongation before rupture when tested in specimen bars. Should not the same quality be recognized in the case of the full-size members of the structure? A tension member that stretches twice as much as another before rupture is just as much better as if the difference were due only to the ductility of the metal used. If the eye-bar will stretch four times as much as the built-up member, the result is the same as if the metal possessed four times the ductility, assuming that the two bars have the same ultimate strength. To put the same thing in another way, let it be considered as a question of resilience. It was because of his recent studies of the resilience of engineering materials that the speaker's attention was called to this matter of elongation of built-up tension members, as elongation is essential to resilience under tensile stress.

What may be called for convenience "ultimate resilience" is really the work in foot-pounds required to break the member. In the case of Specimen No. 10, Series A, the maximum load was 205 000 lbs., the elongation (1.56 ÷ 12 ft.), and the resilience 205 000 × 1.56 ÷ 12 = 26 650 foot-pounds, nearly.

With an eye-bar of the same length and strength and four times the stretch, the resilience would be four times as great, or 106 600 foot-pounds, nearly. To illustrate: A weight of 1 000 lbs. falling 25 ft. would

break the built-up member, while the same weight would have to fall nearly 100 ft. to tear the eye-bar in two, although each has the same strength under a static load. Mr. Russell.

In applying this idea to railroad bridges, it may be said that two steel bridges could be built, each to carry the same breaking load, and yet one would be destroyed by a collision which the other would sustain. The latter would suffer some distortion perhaps, but not utter failure.

As long as strains are kept below the elastic limit, the built-up member is as good as the solid bar. When the real crisis comes, however, it is the ultimate strength of the structure that counts, and the strains will exceed the elastic limit. In the same way the "ultimate resilience" is what saves the bridge when the collision occurs; moreover, the ultimate resilience of one member of a bridge may sometimes determine the fate of the whole structure.

If it is admitted that there must be a certain resilience in the bridge member, then it will be seen that to obtain it with riveted work there must be about four times the section that would be required with an eye-bar. To make up for the failure to stretch, the built member must be given four times the strength of the eye-bar. An eye-bar having an ultimate strength of but 50 000 lbs. would withstand as great a blow as would the built-up bar of the same length with an ultimate strength of 205 000 lbs., such as bar No. 1, Series A.

In conclusion, the point is that engineers, in substituting built-up for solid members, where they find it necessary, should do so with the full knowledge of what they are sacrificing in the way of resilience and in the way of safety. It may be true that in some cases the gain in stiffness due to excess of metal in built-up members would compensate for the loss of resilience. This would not be true in all cases, however, and stiffness may often be too dearly purchased at the price of resilience.

George S. Morison, Past-President Am. Soc. C. E.—There are one or two points about this paper to which attention should be called. In the first place, on Plate IV, angles are shown which have been tested, these angles being connected by rivets with flats through which the strain is applied. Although in some instances the flats were riveted to but one leg of the angle, and in other instances to both legs of the angle, the plane of the flat always corresponded to or was parallel to the plane of one leg, and was in no instance a mean between the planes of the two legs. It is quite possible that if the position of the pulling plane had been made an average between those of the two legs to which it was riveted, a different result might have been obtained. Mr. Morison.

There is another matter, however, to which attention should be called. The speaker had never seen any tests made, or any account of tests, which gave a correct measure of the elastic limit and ultimate

Mr. Morison. strength of a riveted member and of precisely the same member without rivets. Take, for instance, a plate 10 x 1 in. This is a pretty good-sized plate, but it can be tested and will give a certain elastic limit and a certain ultimate strength. Then take another piece of the same plate, drill a lot of rivet holes on some systematic plan and test this second specimen. What will be the ultimate strength and the elastic limit of that second test as compared with the original test? The ultimate strength will undoubtedly be lower in a proportion which bears some relation to the number of rivets. The reduction will probably be considerably less, and the elongation will probably be less. But it is not clear that there will be much, if any, difference in the elastic limit; that is, not in the elastic limit per square inch, but in the elastic limit of the entire plate. If, furthermore, these rivet holes are filled solid with rivets, it is probable that the elastic limit will be about the same as that of the solid plate, that the modulus of elasticity on the net section will be raised, and that of the gross section will not be reduced. All this has a very important bearing in the proportioning of riveted tension members, and may justify the practice in positions in which riveted tension members must be used, like the flanges of a plate girder, in making a very much less reduction for rivet holes than is now considered necessary.

All the results given in the paper are apparently figured on the net section of the riveted member and not on the gross section. If applied to a gross section proportional results would be very different. The real advantage of an eye-bar is not that it is the strongest form in which metal can be put, but that it is the form in which a minimum section can be reserved for the greatest proportion of the entire length of the member. In any form of riveted tension member, proportioned on net section, a considerable excess of material is carried through the whole length of the member. In an eye-bar the excess is confined to the heads. If the relative weights of riveted tension members as compared with eye-bars are calculated, the difference will be found to vary greatly with the length of the members. There is certainly an economical limit to the length of eye-bars. An eye-bar should not be used when the distance between the heads is only a small multiple of the diameter of the head. It is better to use a solid piece of metal and bore the two pin-holes.

The paper makes some very valuable suggestions, and it may be made a basis for careful studies and estimates.

Mr. French. JAMES B. FRENCH, Assoc. M. Am. Soc. C. E.—The author compares riveted tension members and eye-bars on the basis of "the percentage ratios of the developed strength to the strength of specimens." While this is a natural basis of comparison, it hardly does full justice to the riveted member. Investigations have, so far, failed to establish any definite relation between the thickness of the metal, or the area of

the cross-section, and the ratios referred to, but it is well known that, starting with the same grade of steel in the ingot, the strength of the finished material will largely depend on the amount of work it receives in the rolls. In other words, if the same ingot of steel be rolled partly into eye-bar flats, and partly into the smaller shapes that would make up a riveted member of equivalent area, the latter shapes will have the greater strength per square inch; or, if the strength per square inch in the two members is to be the same, a higher grade of steel will have to be used in the eye-bar than in the riveted member. Mr. French.

As compared with eye-bars, therefore, riveted tension members of equivalent section are made up of more thoroughly worked material; their manufacture in parts allows better inspection and makes the discovery of flaws more certain; for the same strength per square inch, a grade of material can be used in them less sensitive to slight irregularities in manufacture, and their form, if properly designed, is equally efficient in the development of the strength of the material of which they are made.

It would seem, therefore, not only justifiable to use as high unit stresses in these riveted members as in eye-bars, but if any preference is to be shown toward either, the former would seem entitled to it.

J. P. Snow, M. Am. Soc. C. E.—With reference to the remarks of Mr. Russell, it is well to consider if there is not a radical difference in the value of elongation when exhibited by a test specimen and by a finished bridge member. The percentage of elongation in a test specimen is a valuable indication of the ductility of the material. It is the measure of the capacity of the material to stand abuse in the process of manufacture and afterwards. Having secured good material, however, a large elastic elongation in the finished member has certain disadvantages. It gives rise to a large deflection, and the elastic recovery of this deflection produces a large vibration, which leads to motion and wear among the parts of a structure. Rigidity in a bridge is just as much an element of usefulness as ultimate strength; perhaps more, if the bridge will not fail under its greatest loading. Bridges are not designed to break down, but to act satisfactorily in use under loads; and if the strains produced by these loads are well within the elastic limit, it is of but little account what happens when a member is tested to destruction. Under ordinary working strains an eye-bar elongates like an ideal test specimen, very nearly uniformly for each unit of its length. A riveted member on the contrary elongates principally at its weak sections, while between these weak sections the elongation is but little. Hence for a given load within the elastic limit a riveted member as a whole will elongate much less than an eye-bar. This greater rigidity of riveted work, due partly to its smaller elongation, is what makes a light riveted bridge more satisfactory than a light eye-bar bridge. Mr. Snow.

Mr. Snow. If it is claimed that riveted work is more liable to fail from a sudden blow than eye-bar members, the answer is that experience does not show this to be the case. Theoretically the high modulus of elasticity due to the small elongation of a riveted member would lead to small resilience, but resilience does not fully measure the usefulness of a bridge structure. It may be the governing factor in considering the value of a spring, but the old-fashioned ideas of the necessity of springs and cushions about a bridge were abandoned long ago.

The author's experiments show that, when designed with a proper regard to symmetry, a riveted member may be figured for the same unit strains as an eye-bar in a similar position. It would seem from this that the practice advocated by some engineers of assessing plates and shapes, or in other words riveted work, with lower unit strains than eye-bar members, is not warranted. Indeed, for strains well within the elastic limit, it is fairly questionable if the assessment should not be the other way, particularly in the case of plate girders.

In Series E of the experiments, mention is made of the distortion of rivet holes near the end of connecting plates, causing the rivets to be loose and giving them the appearance of shearing. This, of course, did not occur within the limit of ordinary working strains, still it probably indicates the weakest point in ordinary riveted work. Where a member is connected by a long line of rivets the load transmitted to them must be very unequal, the elongation of the member having a tendency to throw much less load on those next the end of the member than on the others. Moreover, if there is any jar or side vibration in the member, the first rivet in the gusset plate must resist nearly the whole of it. These considerations show the necessity of using as many lines of rivets in the connection as are admissible and would lead to connecting both legs of an angle if no other reason demanded it. This unequal distribution of load among rivets in long lines marks the proper division between riveted and pin work. Where the load is so great that long lines of rivets are necessary, it is better to concentrate it on a pin. If all rivets could be machine driven, large sizes could be used and their number lessened; but in field connections, where hand driving is unavoidable, it is not safe with ordinary workmen to use rivets much larger than $\frac{7}{8}$-in. diameter.

# CORRESPONDENCE.

GEORGE H. THOMSON, M. Am. Soc. C. E.—The following is offered more as a contributive experiment than as a discussion. Mr. Thomson.

The experiment was a full-size test of a built-up eye-bar, supposed to represent a forged eye-bar of a section of 30 sq. ins.

The bar was composed of one plate $31\frac{1}{2}$ ins. $\times$ 0.96 in. $\times$ 23 ft. 4 ins., with holes for 10-in. pins, 20 ft. centers, and with thickening plates $31\frac{1}{2}$ ins. $\times \frac{3}{4}$ in. $\times$ 4 ft. 5 ins., two at each pin hole.

The main plate, intended to be 1 in. thick, being but 0.96 in. thick, after deducting a rivet hole $1\frac{1}{4}$ ins. in diameter, was found to have a net section of 28.675 sq. ins. instead of 30 sq. ins.

The main plate had defects along the edges.

Each eye was covered by twenty-eight $1\frac{1}{4}$-in. rivets, so disposed as to weaken the bar by one rivet hole.

Between the pin-plates and along the center of the bar $1\frac{1}{4}$-in. holes, spaced 12 ins. centers, were drilled, making the total number of these holes fifteen.

All holes were solid drilled.

The experiment was made to test the built-up eye, rather than the strength of the bar or plate itself.

The main plate, prior to its being manufactured into an eye-bar, was said to possess the following properties: Tensile strength, 71 500 lbs. per square inch; elongation in 8 ins., 20 per cent.

The elastic limit and reduction of area at fracture were not given.

The chemical analysis was said to be: Carbon, 0.28; phosphorus, 0.065; manganese, 0.37.

The thickness of the head of the manufactured bar was 2.46 ins.

The built-up bar being too large in the head to go into the testing machine, short links were used to connect the bar to the main pins of the machine. Upon the first trial these links failed, and upon heavier ones being provided the bar was finally tested at Phœnixville, Pa., May 28th, 1892, with the following results, as reported by F. H. Lewis, M. Am. Soc. C. E., who had charge of the inspection: Total tensile strength, 1 454 000 lbs.; tensile strength per square inch, 50 700 lbs.; total elastic limit, 1 010 000 lbs.; elastic limit per square inch, 35 220 lbs.; elongation in 14 ft., 3.9 ins.; elongation in entire bar, 4.625 ins.; elongation of each pin hole, 0.09 in.; fractured area, 27.606 sq. ins.; greatest elongation in 12 ins., 3 ins.; reduction of area at fracture, 3.73%; character of fracture, granular.

The elongation of the $1\frac{1}{4}$-in. holes laid along the center of the bar averaged 15.4%, the minimum being 12.8%, and the maximum 18.4 per cent.

Mr. Thomson.

The location of the fracture is shown in Fig. 7.

The action of the mercury gauge was peculiar, advancing quickly in jumps of half an inch or thereabouts, and then stopping or receding a little.

The time required for the test was ten minutes.

After the conclusion of the experiment, two pieces, about 1 in. square, were cut from the middle of the bar, and gave the following test results: Tensile strength, 71 380 and 71 800 lbs. per square inch; elongation in 8 ins., 1.42 and 1.20 ins.; percentage of elongation, 17.75 and 15; reduction of area, 37.34% and 31.07%; character of fracture in first specimen, 75% granular and 25% silky; character of fracture in second specimen, 5% granular and 95% silky.

The $\frac{3}{4}$-in. thickening plates were of steel, possessing superior quality, with not high properties, about 64 000 lbs. ultimate.

There were no evidences of rivet failure.

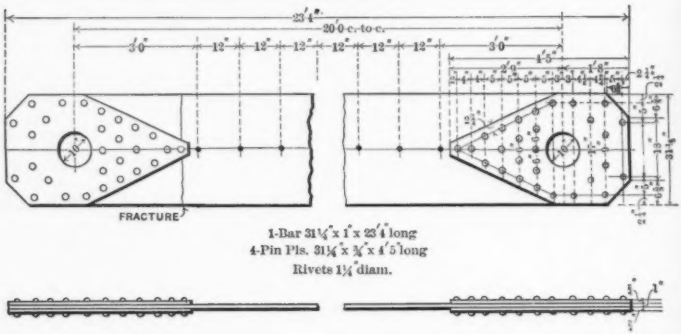



FIG. 7.

Mr. Broomall.

C. M. BROOMALL, Jun. Am. Soc. C. E.—A most instructive lesson can be obtained from a study of the photographs accompanying the paper. An examination of the position of the line of rupture shows with what unerring certainty it chooses the weakest part, usually a line of rivets. From a study of these photographs, the faults in a particular design are evident, and can be corrected. A point worthy of attention is shown in Plate IV, where the very interesting case of the combined effect of direct stress, shear and bending moment determines the rupture. This is a combination which is usually not taken account of in the calculation of riveted joints. Owing to the shearing pieces not being infinitely thin, the pins are subjected to a bending moment, which is transferred to the plates, bending them. In the photographs it is seen that in all cases the cause of the rupture is this combination of the direct tension with the bending. The rupture oc-

curs at the end of the connections since their stiffness prevents excessive bending everywhere, except at the ends. In the bars in question the reason for the rupture occurring at the inside end seems to be due to the fact that the test pieces are not as heavy as the pieces to which they are connected, and therefore the greatest amount of bending, and consequently the rupture, occurs at the inside end of the connections. Mr. Broomall.

J. C. MOSES, Jun. Am. Soc. C. E.—Tests made some years ago on eye-bars showed that bars having a row of holes drilled on the center line of bar were much stronger than similar members with the holes drilled on a line parallel to, but not coincident with, the center line. If the angles described by the author in Series E, Nos. 43 and 44, be considered as opened out into flat bars, the following computation may be made (see Fig. 8): Mr. Moses.

Let $P$ = total stress.

$A$ = net area of section = 5.62 sq. ins.

$p = P \div A = P \div 5.62$.

$d$ = eccentricity of section through hole.

$f$ = maximum fiber stress from bending moment $M$, due to eccentricity of section.

$$M = Pd = 0.3\ P = \frac{f\ I}{Y_1} = f\ \frac{21.3}{3.61} = 5.88\ f.$$

$$\text{Therefore } f = \frac{0.3\ P}{5.88} = \frac{0.3 \times 5.62}{5.88}\ p = 0.29\ p.$$

$$\text{Maximum unit stress} = f + p = 1.29\ p.$$

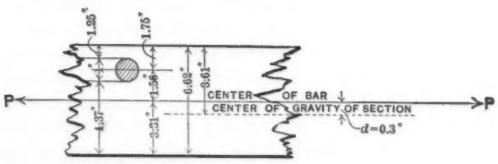



FIG. 8.

If the unit stresses given in Table No. 17 for these two cases be multiplied by this factor, values will be obtained much more nearly equal to those derived from the test specimens. The decreased unit strength of all the tension members described in Series A, B, C and E may be ascribed to this eccentric punching, the results being modified by fixed end connections and support of lattice bars, etc. By making allowance for this eccentricity when figuring the required net sections of angles, more correct results will be obtained than by the customary method of using a lower unit stress for shapes.

If two symmetrically placed holes be punched out of the angle

Mr. Moses. illustrated in Fig. 8, the net section will be 4.62 sq. ins., and the maximum unit stress $5.62 \div 4.62 = 1.22\,p$. Theoretically, therefore, the angle is stronger with two holes punched out of it than with one. The actual distribution of stresses no doubt varies from the uniform distribution assumed above; but the writer would suggest as an improvement on present practice, that at least two holes should always be deducted when obtaining the net sections of angles.

In figuring net sections, it is generally assumed that if one hole only need be deducted at the first point, two may be taken out at the next section without further reducing the strength of the member. This assumes that a rivet is worth as much as the area of metal cut out for the rivet hole. This is not true for the unit stresses ordinarily used, but is, however, a very common error of design.

In the author's discussion of the results of tests *E*, considerable importance is attached to the comparison of the ultimate strength of angles connected by one leg and by both legs. It seems to the writer that the proper basis of comparison is the elastic limit. The ultimate strength is of questionable value and had better be omitted in tests of this kind. A comparison at stresses below the elastic limit would be better still, if practicable.

The high elastic limits apparently obtained for the built members are, perhaps, due to the first elongations taking place at the rivet-holes only. They would, therefore, not be detected as readily as with a standard test piece. The author's explanation of the phenomenon would be of interest.

Mr. Greiner. J. E. GREINER, M. Am. Soc. C. E.—Mr. Russell has brought forward the question of resilience in connection with railroad bridges, and states that: "If it is admitted that there must be a certain resilience in the bridge member, then it will be seen that to obtain it with riveted work there must be about four times the section that would be required with an eye-bar," and that "engineers, in substituting built-up for solid members, * * * should do so with full knowledge of what they are sacrificing in the way of resilience and in the way of safety." In other words, Mr. Russell believes that a bridge whose tension members are eye-bars will be better fitted to withstand the shock of a collision than a bridge having built-up tension members of the same net section. This conclusion, while it may appear correct from the standpoint of resilience, is not in accord with modern bridge practice nor with experience. Mr. Snow answers Mr. Russell very neatly when he says that resilience does not fully measure the usefulness of a bridge structure, although it may be the governing factor in considering the value of a spring. Modern railroad bridges must be made stiff and vibration eliminated as far as possible, or they will soon become rattle-traps under the heavy motive power now in use. Springing boards are no longer admissible except in cases of highway

bridges, and in such cases it cannot be said that their great resilience has always prevented them from being knocked down by an ordinary farm wagon. Mr. Greiner.

If Mr. Russell will examine the make-up of the sections tested, he will see that while the elongation is given from back to back of pin holes, the stretch actually occurred within the very short space where the main angles were not re-enforced by pins and batten plates and therefore the resilience of these members is not nearly so small as would appear.

Mr. Morison calls attention to the relative strength of a solid plate, one with open drilled holes, and one with these holes filled with rivets, no tests of which have been published so far as known. The suggestion made has a very important bearing on riveted tension members, and should be carried out. There are times when the eye-bar, as ordinarily made, is too small for use, and it becomes desirable to build up a bar of two or more plates. These plates being riveted together, the question raised is whether it will be necessary to consider the actual net section of such a bar, the gross section, or a section somewhere between the two. The test given by Mr. Thomson is somewhat on this order and shows a considerable falling off in ultimate when compared with specimens.

The writer believes that tests of a somewhat similar nature have been made by C. C. Schneider, M. Am. Soc. C. E., with quite different results, but the results not having been published, they cannot be discussed.

The distortion of rivet holes near the ends of connecting plates in Series E, causing the end rivets to become loose and shear, while the balance remained tight, indicates clearly that in all riveted connections, the rivets should be grouped closely rather than in long lines, as by so doing there is more chance for all of them doing some work. This point is discussed by Mr. Snow, and his discussion is worthy of much consideration.

The elastic limit of the built members was determined by the dropping of scale and the action of the gauge of the testing machine as noted in the paper. It was noticed that at the instant the beam dropped, small scales would fall from the members, usually, though not always, near the part where rupture finally took place. When scaling once began, it covered a large surface rapidly and almost before the observer could identify the particular spot where it commenced. Mr. Moses' explanation of the high elastic limit is probably the correct one.

It is to be hoped that the suggestions brought out in the discussion will cause someone who has the opportunity to continue the tests, as their practical value must be admitted.